Structural Health Characterization of an Old Riveted Iron Bridge By Remote Sensing Techniques

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Abstract

A 45 m long single span truss road bridge, built in 1930 in Southern Italy, is still serving the local community while no regular maintenance operations have been performed since its construction. In order to estimate its actual static and dynamic behavior, to predict its load carrying capacity and to preliminary estimate its remaining service life, an extensive campaign assessment, within an available short time frame, is carried out by Terrestrial Radar Interferometry. The maximum deflection registered, in the worst reproduced loading scenario, is about 3 mm at mid-span, while the first frequency of vibration registered in the vertical plane is 3.4 Hz. By a suitable Finite Element Model, calibrated by field tests results, the overall mass of the structure (201 t) and the moment of inertia of the deck cross section at mid-span (0.42 m^4) are retrieved too.

1. Introduction and Scope of Work

The observational method (Peck, 1969) plays a key role in the design and management of large engineering structures and can provide primary information in every design step, from the preliminary design up to the executive one (Nicholson et al., 1999). During the last few years, a significant development of several innovative technological solutions has been observed which represent, nowadays, a great opportunity for the observational method. In the science of "measure" the main recent innovation is represented by "remote sensing systems" (Mazzanti, 2012), whose strong capabilities are today extensively demonstrated.

One of the most effective remote sensing system, used for the characterization of structural performance of roadways and railways infrastructures, is the Terrestrial Radar Interferometry (Bernardini et al., 2007; Hanssen, 2001). Thanks to its undeniable advantages (such as quick installation, high resolution of investigation, etc.), it has been extensively applied over the last decade to check the structural behavior of bridges and towers under static and dynamic loadings, thus providing excellent results if compared to those obtained using conventional monitoring techniques (Pieraccini et al., 2008; Atzeni et al., 2010; Gentile & Bernardini, 2010; Cunha et al., 2001; Mazzanti et al., 2014).

This paper describes the application of Terrestrial Radar Interferometry for the assessment of the structural performance of an historical old riveted roadway bridge (made of composite steel sections and a reinforced concrete deck) as imposed by the Italian Code of Practice (Italian Design Standard, 2008).

The bridge, designed and constructed soon after the First World War in South Italy, belongs to the Municipality of Amantea (Figure 1) and it is still opened to the local daily traffic, linking the historical center of the city to the main expressway.

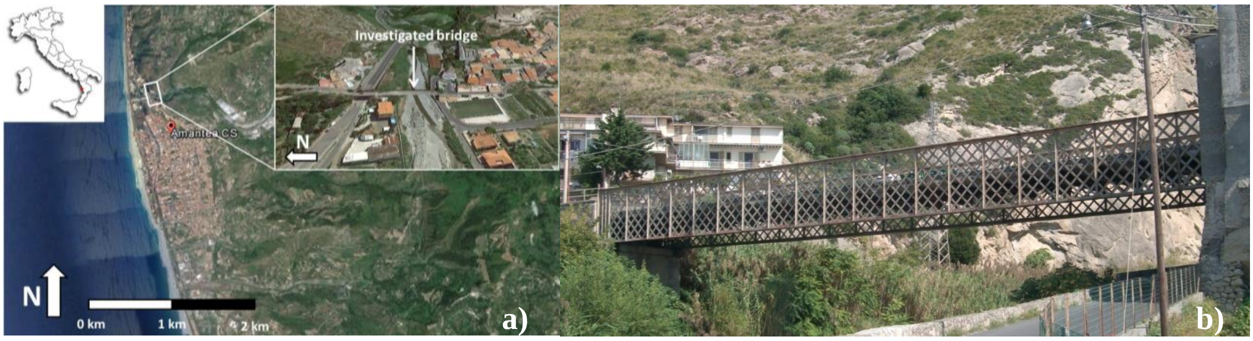

Figure 1. Location of the investigated bridge on satellite image (a) and overall view of the Old Riveted Steel Bridge analyzed in this paper (b).

Because no maintenance operations have been properly carried out during the past decades and, due to its highly corroded steel frames, an urgent assessment of its structural capacity is necessary.
Due to limited financial resources and a short-time frame available to check the structure, Remote Sensing Techniques (Mazzanti, 2012; Tapete et al., 2013; Mazzanti et al., 2014) are preferred to conventional contact monitoring systems to get results in only one day field work. Specifically, a Terrestrial Interferometric Radar system is adopted and temporary installed on site during the field investigations, recording vertical deflections of the whole structure under both static and dynamic loads, applied to simulate ordinary site conditions (Technical Guide SETRA, 2006). Experimental tests (Radomsky, 2002), in fact, are the most reliable method to obtain the dynamic properties (natural frequencies, mode shapes and damping ratios) of existing structures and are used to validate numerical models implemented for their back analysis (Caruso, 2013; Caruso, 2014).
The experimental program, developed in this work, is devoted to the structural characterization of the bridge both in terms of vertical deflections (static tests) and vibration modes (dynamic tests). Static tests are performed through several loading phases conducted using ordinary vehicles eccentrically placed, with respect to the deck cross-section and distributed along the span in order to maximize flexural, as well as, twisting effects. Dynamic tests are performed, on the other hand, allowing the transit of pedestrians and ordinary vehicles crossing the bridge at different speeds with and without obstacles placed at mid-span (simulating the dynamic impact force due to surface irregularities).
The main scope of the work is to give an estimation of the capacity of the existing structure and to determine, within an acceptable range of accuracy, its remaining service life and, if necessary from back analysis calculations, proposing the main retrofitting procedures to be addressed to improve its safety.

2. Structure Description and Main Features

The roadway bridge is made of two longitudinal steel truss girders restrained together by transverse beams 2 000 mm spaced apart, along its entire span length of 41.60 m. In Figure 2, the main dimensions of the bridge are computed on the 3D point cloud (achieved by a suitable Terrestrial Laser Scanner Survey) to be used as input data for the setting up of the mathematical model of the bridge. Each structural member has a composite section field-assembled through the use of double angle profiles and plates of different thickness tight together by hot hammered riveted bolts, according to the methodologies widely implemented at the beginning of the 20th Century in Europe for the construction of important steel structures, like roadway and railway bridges.

Each steel truss girder is 3 500 mm high, equipped with top and bottom cords connected together by vertical members every 2 000 mm and inclined steel elements running between these cords, able to transfer axial forces while keeping them horizontally aligned.
The reinforced concrete deck is placed, approximately, at girder mid-height, restraining the overall bridge against lateral actions, having an overall length of 45 m, as measured between expansion joints. A proper bracing system, placed underneath the deck, connects the transverse beams between each other, strengthening the structure against horizontal wind forces.

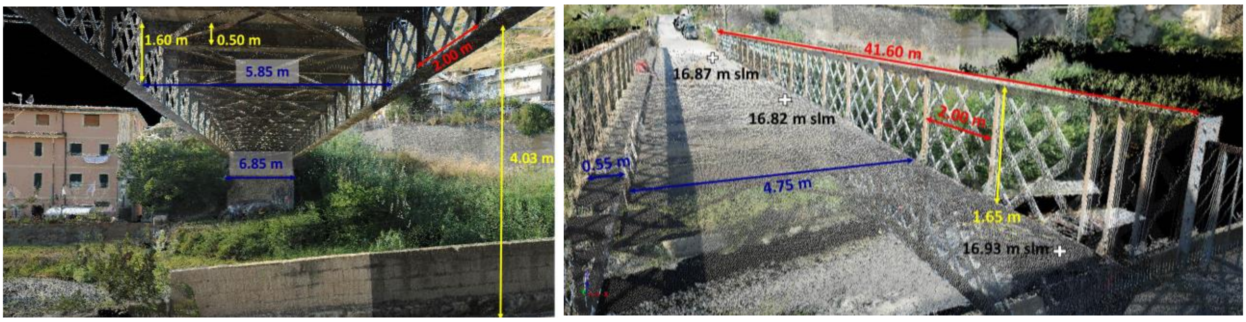

Figure 2. Main dimensions of the investigated bridge, measured on the 3D point cloud achieved by Terrestrial Laser Scanner Survey

Longitudinal girders rest, at their ends, over steel plates which represent the bearing devices placed on top of masonry abutments, used to transfer the vertical components of the end reactions to the foundation soil.

3. Performed activities

Field investigations of the bridge are performed on 7th August 2014 by Terrestrial Radar Interferometry for static and dynamic tests. Specifically, an IBIS-S (by IDS S.p.A.) Interferometric sensor, able to simultaneously measure the displacements along the instrumental line of sight (LOS) of a large number of points along the structure by high sampling frequencies (up to 200 Hz), is temporary installed beneath the bridge on the road which underpasses the analyzed structure. Thanks to the vertical clearance of $4.60\ m$, available between the road and the bridge deck, the instrumental field of view is able to cover almost 80% of the span length. The monitoring platform is also equipped with a triaxial velocimeter sensor (Syscom MS2003+) installed on the roadway at mid-span of the bridge for the calibration of interferometric measurements.

3.1 Static tests

Two tests are performed in order to detect the deflection of the bridge when loaded with ordinary vehicles:

i) Application of a point load at deck mid-span (loaded pick-up of about 25 kN, Figure 3a). The test is performed applying the load twice for the deformation recovery analysis.
Duration: 11 minutes.

ii) Ordinary vehicles uniformly and progressively distributed along the bridge deck and a loaded pick up (25 kN) finally placed at mid-span (Figure 3b). The test simulates the worst loading scenario according to local traffic conditions.
Duration: 63 minutes.

During the above testing phases, loading and downloading sequences are constantly monitored by the interferometric system with a 10 Hz sampling frequency and a spatial resolution of 0.5 m along the instrumental line of sight (LOS).

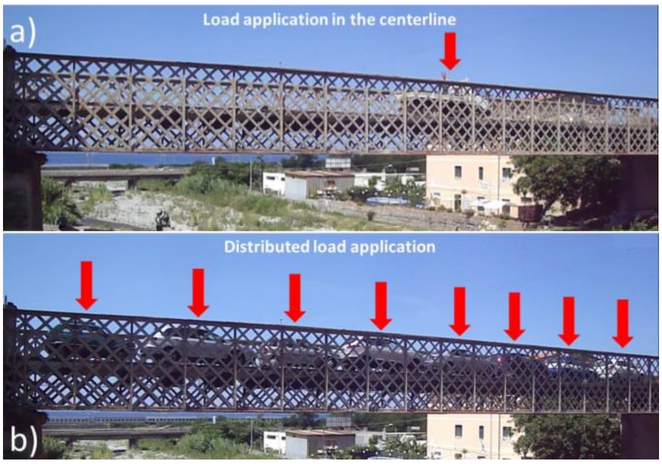

Figure 3. Schematic sketch of the static tests.

3.2 Dynamic tests

Five tests are performed in order to detect the dynamic behavior of the bridge in different service conditions:

i) No traffic load (only environmental noise like the wind acting on the truss beams).
Duration: 5 minutes.

ii) Ordinary vehicles moving at different speeds (no obstacles on the roadway).
Duration: 9 minutes.

iii) Ordinary vehicles moving at different speeds, with an obstacle on the roadway for the amplification of the resulting force effects (simulating the irregularities of the roadway).
Duration: 11 minutes.

iv) Group of pedestrian randomly running on the bridge deck.
Duration: 3 minutes.

v) Cyclic force (people jumping on the roadway at a frequency close to the frequency of vibration in the vertical plane).
Duration: 7 minutes.

During all tests, the structure is constantly monitored by the interferometric system with a 100 Hz sampling frequency and a spatial resolution of 0.5 m along the instrumental line of sight (LOS).

4. Achieved results

4.1 Static tests

By the interferometric survey, vertical deflections of the structure are properly recorded. In Figure 4a and Figure 4b are reported, respectively:

i) deflections of the whole visible part of the bridge following the point load at mid-span. The maximum registered deflection (at mid-span) is about 0.9 mm;

ii) deflections at mid-span following the progressive application of distributed loads during the different phases of the test. The maximum registered deflection is about 3mm. Phase 4, shown in the same graph of Figure 4b, clearly illustrates that the maximum vertical displacement, reached at the end of Phase 3 (soon after the placement of the loaded pickup), is fully recovered by the structure highlighting its perfect elastic behavior.

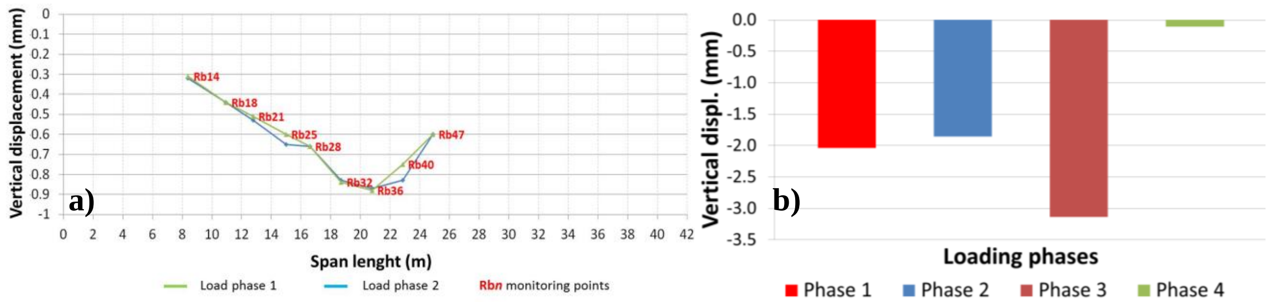

Figure 4. Vertical deflection of the whole visible part of the bridge from the Interferometric monitoring point (a) and vertical deflections of the bridge at mid-span, plotted against the loading sequence made of ordinary vehicles and a loaded pick-up. The total deformation is fully recovered once traffic loads are removed (b).

4.2 Dynamic tests

Results achieved by Interferometric survey during dynamic tests are available for all the visible sectors of the bridge deck along the instrumental line of sight.

Results are achieved both in time and frequency domain, allowing the calculation of the damping ratio and the main modal parameters of the analyzed structure.

The following graph (Figure 5) displays a frequency spectrum with the identification of the First Frequency of Vibration in the vertical plane at bridge mid-span ($f \approx 3.4$ Hz, with a corresponding Period of Vibration of $T \approx 0.290$ s). Specifically, it refers to the dynamic test performed using ordinary vehicles travelling at a constant speed of 20 km/h and running over a wooden ramp placed at deck mid-span. Basically, all performed tests give the same frequency peaks while their amplitudes vary as a function of the applied load and moving speeds.

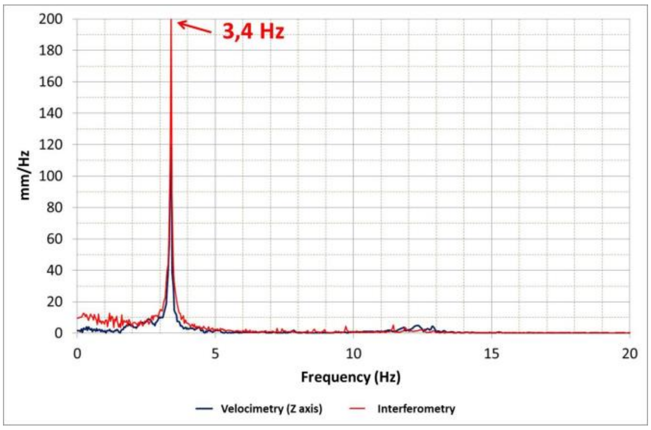

Figure 5. Structural response of the bridge at mid-span in the frequency domain achieved by Interferometric monitoring during the test with ordinary vehicles travelling at 20 km/h, with an obstacle on the roadway. A comparison with the results achieved by the velocimeter sensor on the vertical plane is also reported for data validation.

Data recorded and plotted in the time domain were used to work out the Equivalent Viscous Damping Ratio ξ of the structure, which helps to detect anomalies in the structural behavior, analytically determined by applying the following equation (Ranieri, 2009):

$$\xi = \frac{2\pi}{\delta} = \frac{\delta}{\sqrt{\delta^2 + 4\cdot\pi^2}} \qquad (1)$$

through the implementation of the Logarithmic Decrement Method, based on the following governing equation:

$$\delta = ln\left(\frac{U_1}{U_2}\right) \qquad (2)$$

Where
δ: Logarithmic Decrement
U_1: Displacement amplitude at time instant t_1
U_2: Displacement amplitude at time instant t_2

Damping ratios, carried out for the main dynamic experimental tests, result in an average value of 0.60 % which is very much the same of the values suggested by highly recognized international Codes of Practice (CEB Bulletin No.209) for bolted steel-concrete composite structures ($\xi = 0.40 \div 0.63\%$).
Figure 6 shows the free oscillations of the bridge as registered by the Interferometric sensor soon after the passage of an ordinary vehicle without obstacle on the roadway; this data is used to determine the displacement amplitude at different time intervals during the free-standing motion of the bridge.

By extracting the picks from the previous graph and working out the main amplitudes, as plotted in the time domain, the equivalent damping ratio is then carried out based on a population of n.10 pick values as shown in Table 1.

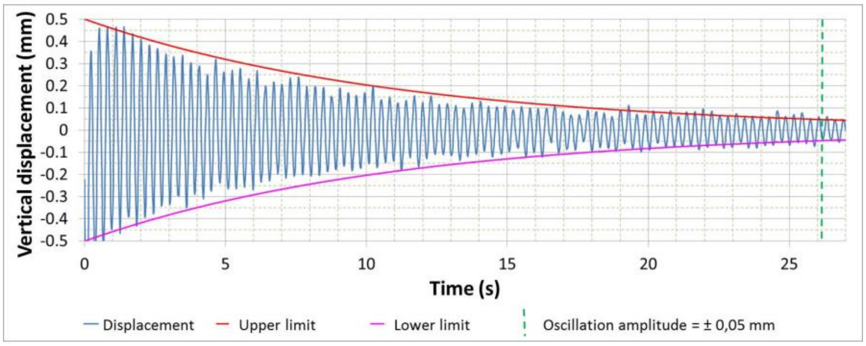

Figure 6. Graph illustrating decreasing vertical displacement amplitudes versus the time of the mid-span section of the bridge soon after the passage of a loaded pick-up travelling at a constant speed of 50 km/h over the carriageway.

Table 1. Calculation of the Equivalent Viscous Damping Ratio of the bridge soon after the passage of a loaded pick-up travelling at a constant speed of 50 km/h.

V = 40 km/h									
Population	Nr. of Peaks	Time Instant (s)	Displacement (mm)	Time Instant (s)	Displacement (mm)	Logarithmic Decrement	Damping Ratio		
	n	t1	U1	t2	U2	δ	ξ (dim)	ξ (%)	Average Value (%)
1	1			4.399	0.204	0.0293	0.00467	*0.47*	
2	2			4.692	0.198	0.0587	0.00933	*0.93*	
3	3			4.985	0.192	0.0440	0.00700	*0.70*	
4	4			5.279	0.187	0.0391	0.00622	*0.62*	
5	5	***4.106***	***0.210***	5.572	0.181	0.0367	0.00583	*0.58*	***0.60***
6	6			5.865	0.176	0.0352	0.00560	*0.56*	
7	7			6.158	0.171	0.0342	0.00545	*0.54*	
8	8			6.452	0.166	0.0335	0.00533	*0.53*	
9	9			6.745	0.161	0.0330	0.00525	*0.53*	
10	10			7.038	0.156	0.0326	0.00519	*0.52*	

5. Bridge Finite Element Model

The finite element model (Chapelle & Bathe, 2003; Bathe, 1996) is built by setting the structure geometry and material as the only input data. The mathematical model of the bridge has been built using two different widely used finite element software in order to simulate the behavior of the real structure under the combination of elementary load cases.
The whole structural model is made of 1 747 nodes and 3 744 and four-nodes shell finite frame elements with geometrical and material properties of the existing structure.
The metallic single-span bridge is resting over four plates which are entirely corroded and unable to allow longitudinal movements due to thermal effects or applied loads. As a consequence, global external supports are modeled as fixed hinges able to allow only rotations of the main longitudinal steel members within their vertical plane, while restraining horizontal translations.
The concrete deck slab, which represents the traffic roadway, is subdivided in a series of shell elements restrained, at both ends, to the approaching road embankment using hinges. The roadway slab is 1 500 mm longer than the bridge truss girders from both sides, resulting in an overall length of 45 m. This complex system, depicted in Figure 7a, represents the whole structure which has been studied in this work.

As reported during site inspections, the two main steel beams have been designed with top and bottom chords of different cross-sections, following the bending moment diagram. In fact, being the flexural force effect linearly increasing from support to mid-span, additional plates are added to both top and bottom chords in order to increase the flexural stiffness of the sections while keeping the resulting axial stresses below the allowable stress limits.

As per original design, each girder is then divided into four zones which are characterized by chords of different thicknesses (Figure 7b). It has been reflected in the finite element model of the bridge in order to be consistent with the overall design process and to accurately reproduce important features of the real structure.

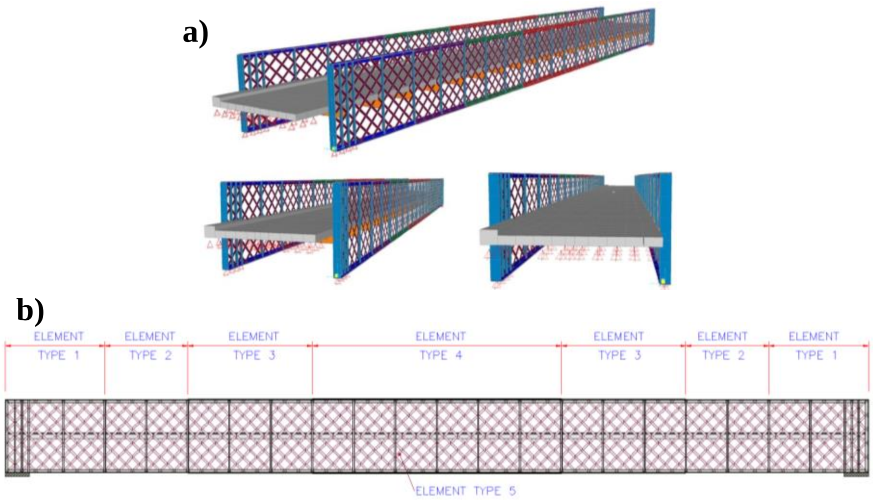

Figure 7. Pictorial representation of the whole structure showing the real dimensions of its single elements (a) and geometrical model of the metallic bridge, showing the main types of elements optimized in the iterative procedure (b).

Cross-sectional areas of the main members, belonging to each of the four zones, are summarized in Table 2. The resulting numerical model has been obtained through an iterative process by increasing, at the end of each step, the cross-sectional area of the main elements. The following table shows the first assigned properties and those resulting at the end of the optimization process, with the involved structural members.

Table 2. Cross-sectional Area of the main structural elements assigned to the numerical model of the investigated bridge.

Structural Element	Initial Cross-Sectional Area (m^2)	Final Cross-Sectional Area (+30%) (m^2)
Element Type 1 *Top & Bottom Flange*	0.0077	0.0100
Element Type 2 *Top & Bottom Flange*	0.0133	0.0173
Element Type 3 *Top & Bottom Flange*	0.0189	0.0246
Element Type 4 *Top & Bottom Flange*	0.0294	0.0382
Element Type 5 *Truss Web Plate*	0.0008	0.0010

The iterative process is over when the first frequency of vibration of the bridge on its vertical plane is approximately close to that recorded during field tests. Because frequency is a function of the mass and stiffness of the structure, getting the right frequency of vibration, within an imposed

tolerance, means that the structure is very well modeled and accurately represents the real bridge. Properties shown in Table2 refer to the first assigned dimensions (as measured on site) and to those obtained at the end of the iterative process, with an increased cross-sectional area of 30%, taking into account for rivets, cover plates and additional nodal plates very difficult to be measured.
The main modal parameters achieved by the finite element model of the bridge, for the first two modes of vibrations, are summarized in Figure 9.

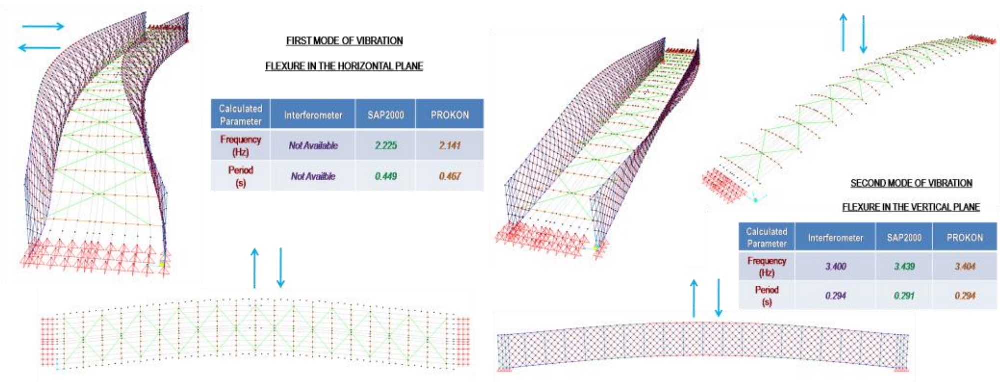

Figure 8. Modal shape and parameters carried out for the first and second global mode of vibration.

By the achieved accurate F.E.M., the overall mass of the bridge (201 t) and the moment of inertia of the deck cross-section at mid-span (0.42 m^4) are also retrieved.
To determine the capacity of the bridge and to give to the chief engineer of the municipality an estimation of its remaining service life, the numerical model, now validated from the results achieved by field tests, is ready to be loaded with superimposed, wind and traffic loads as published in the Italian Design Standard (DM 2008). Furthermore, each elementary load case is affected by partial load factors and then properly combined to carry out the worst force effects at both Serviceability and Ultimate Limit States.

6. Conclusions and Final Remarks

Experimental field tests by Terrestrial Radar Interferometry are performed for the static and dynamic characterization of an old riveted iron bridge. In one only day of field activity, the deflection and the vibration of the structure under different loading scenarios are accurately detected allowing the determination of the actual structural capacity with significant money savings. By a suitable Finite Element Model calibrated with data achieved by field tests, the mass of the structure and the moment of inertia of the deck cross-section at mid-span are also determined.

Results, carried out from the numerical model differ, with respect to those obtained from the field experimental tests, by less than 1 % in terms of displacements and frequency of vibration in the vertical plane so, from the engineering point of view and for the studied problem they can be considered highly accurate, allowing the designer to present to the Client reliable and practical indications for future retrofitting measures.

One of the main insights of this project is the confirmation of the great potentiality and flexibility of Terrestrial Radar Interferometry for the quick analysis of the structural behavior of structures and infrastructures, as already highlighted by Mazzanti et al. (2014) and by previous validation tests through the comparison of interferometric data and data collected by conventional contact monitoring techniques (i.e. velocimeters, accelerometers etc.) (Pieraccini et al., 2008).

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